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Frontier of Continuous Structural Health Monitoring System for Short & Medium Span Bridges and Condition Assessment

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Abstract:

It is becoming an important social problem to make maintenance and rehabilitation of existing short and medium span(10-20m) bridges because there are a huge amount of short and medium span bridges in service in the world. The kernel of such bridge management is to develop a method of safety(condition) assessment on items which include remaining life and load carrying capacity. Bridge health monitoring using information technology and sensors is capable of providing more accurate knowledge of bridge performance than traditional strategies. The aim of this paper is to introduce a state-of-the-art on not only a rational bridge health monitoring system incorporating with the information and communication technologies for lifetime management of existing short and medium span bridges but also a continuous data collecting system designed for bridge health monitoring of mainly short and medium span bridges. In this paper, although there are some useful monitoring methods for short and medium span bridges based on the qualitative or quantitative information, mainly two advanced structural health monitoring systems are described to review and analyse the potential of utilizing the long term health monitoring in safety assessment and management issues for short and medium span bridge. The first is a special designed mobile in situ loading device(vehicle) for short and medium span road bridges to assess the structural safety(performance) and derive optimal strategies for maintenance using reliability based method. The second is a long term health monitoring method by using the public buses as part of a public transit system (called bus monitoring system) to be applied mainly to short and medium span bridges, along with safety indices, namely, “characteristic deflection” which is relatively free from the influence of dynamic disturbances due to such factors as the roughness of the road surface, and a structural anomaly parameter.

Keywords: condition assessment, short & medium span bridge, structural health monitoring(SHM), long-term data collection, system, maintenance, bridge performance, information technology, loading vehicle(public bus), in-situ loading.

1. INTRODUCTION

In the field of bridge management engineering, a great deal of decision making often depends on the assessment and experience of the domain experts in related fields, such as professional experience, knowledge on bridge management, etc. Then an important parameter in management for existing bridges is the remaining life assessment that is crucial as a kernel of the systematization for bridge maintenance throughout service life including economic analyses referring to initial cost and assessment technologies. In bridge structures there are many different unforeseen conditions because bridges are larger and often need to serve longer more than 100 years, compared with the other products such as electrical, mechanical and systems engineering fields. And they are subjected to diverse types of deterioration mechanism such as corrosion, fatigue, carbonation, alkali aggregate reactions, etc. Therefore it is probable to have some risk for not fulfilling the complete standards of safety so some failure probability is possible. Structural aging, environmental conditions, and reuse are examples of circumstances that could affect the reliability and the life of a bridge structure. Engineers have been visually inspecting, monitoring and proof testing bridges for Centuries. However, presently health and performance are described based on subjective measures that are not universal. In addition, defects, deterioration and damage are not discovered until it is possible to visually observe the signs they exhibit at which time these would have taken their toll on health. These shortcomings impact the timeliness, effectiveness and the reliability in any management decision irrespective of any sophistication in the management process. Moreover, even experienced engineers may find visual signs of defects, deterioration and damage and still not be able to diagnose the causative mechanisms, or their impact on the reliability of the system and global health. The global health of an entire bridge as a system, inclusive of the performance criteria corresponding to every one of the limit-states is actually what is needed for effective management decisions. There are needs of periodic inspections to detect deterioration resulting from normal operation and environmental attack or inspections following extreme events, such as strong-motion earthquakes or hurricanes. To quantify these system performance measures requires some means to monitor and evaluate the integrity of civil structures while in service [1-3]. Then, since the necessity of developing a practical health monitoring system has been point out for detecting deterioration phenomenon as early as possible for bridge maintenance.

The aim of this paper is to introduces a state-of-the-art on not only a rational bridge health monitoring system incorporate with the information and communication technologies for lifetime management of existing short and medium span(10-20m) bridges but also a continuous data collecting system designed for bridge health monitoring of mainly short and medium span bridges. In this paper, although there are some useful monitoring methods for short and medium span bridges based on the qualitative or quantitative information, mainly two advanced structural health monitoring systems are described to review and analyse the potential of utilizing the long term health monitoring in safety assessment and management issues for short and medium span bridge. The first is a special designed mobile in situ loading device (vehicle) for short and medium span road bridges to assess the structural safety (performance) and derive optimal strategies for maintenance using reliability based method. In here, the concept of experimental safety evaluation will be discussed; the loading vehicle for short and medium **span** road bridge testing as well as essential operating experiences gained in practise will be described [4,5]. The second is a long term health monitoring method using a public bus as part of a public transit system (called bus monitoring system) to be applied mainly to short and medium span bridges, along with safety

indices, namely, “characteristic deflection” which is relatively free from the influence of dynamic disturbances due to such factors as the roughness of the road surface, and a structural anomaly parameter [6]. In here, it will be described the details of not only how to assess the bridge condition by public bus vibration measured in operating **on an Ube City bus network** in Japan, as a specific example for verify the system but also what kind of consideration we need to apply the system to existing bridges in overseas country.

2. FUNDAMENTAL ISSUES FOR LONG TERM BRIDGE HEALTH MONITORING

In order to establish the rational long term health monitoring for short and medium span bridges, the important issues are highlighted in this chapter:

“Health” can be defined as the reliability of a bridge structure to perform adequately for the required performance as follows [3,7]:

- Safety(load carrying capacity)
- Serviceability
- Hazards for third party
- Aesthetic appearance & landscape and
- Durability

Figure 1 shows the basic components of a typical Structural Health Monitoring System (SHM) [8]. Monitoring is usually carried out in order to achieve one or several goals such as structural safety evaluation, remaining life prediction, etc. thorough the condition assessment and final diagnosis for a target bridge. Based on a general definition as shown in **Figure 1**, monitoring has to be frequent or continuous observation or measurement of structural conditions or behaviors [9]. The structural health monitoring as an integrated system is the use of *in-situ*, non-destructive sensing and analysis of structural characteristics, including the structural response, for detecting changes that may indicate damage or deterioration of the target bridge for evaluating quantify health and reliability indices [10].

In recently, remote measurement of the bridge behaviors using monitoring system incorporate with the latest information technologies and data collecting system has an essential role in SHM system. The data resulting from the monitoring program is used to optimize the operation, maintenance, repair and strengthening of existing bridges based on reliable and objective data. Detection of ongoing damage can be used to detect deviations from the design performance. Monitoring data can be integrated in structural management systems and increase the quality of decisions by providing reliable and unbiased information. Many structures are in much better conditions than expected. In these cases, monitoring allows to increase the safety margins without any intervention on the structure. Taking advantage of better material properties, over-design and synergetic effects, it is possible to extend the lifetime or load-bearing capacity of structures. A small investment at the beginning of a project can lead to considerable savings by eliminating or reducing over-designed structural elements.

In the bridge structures, a strategic life-cycle management aided by the latest information technologies, such as network-based database system, intelligent health monitoring, multimedia

virtual reality, artificial intelligence, etc. has to be developed that increases of their safety and knowledge and experience improvement.

As the technology issues for SHM, in recently, there are many themes for research about SHM. The most important topics are [11,12]:

- Optic fiber sensing
 - Distributed sensors
 - Point sensors
 - Smart and advanced sensors
- Remote sensing and wireless sensor networks
- Data acquisition
- Data analysis
- Signal processing
- Structural/system identification
- Damage detection/diagnosis and damage localization
 - Forced vibration-based damage detection
 - Wind induced vibration-based damage detection
 - Ambient vibration-based damage detection
- Neuro-computing methods for system identification and damage detection
 - Artificial neural networks
 - Fuzzy logic
- Nondestructive structural condition methods and tests
- Structural modeling and simulation by Finite Element Analysis methods
- Model updating, safety evaluation, reliability and decision making
- Intelligent materials and structures
- Database and management systems
- System integration
- Health monitoring for special bridges
 - Short and medium span bridges
 - Long span bridges
- Inclusion of health monitoring system in bridge management system
- Health monitoring of existing and/or new bridge structures
- Life cycle performance design
- Health monitoring of strengthened bridges with new material or methods like FRP

As it is obvious structural health monitoring is a very vast domain for research and it shows several types of benefits for design engineers and practitioners by better understanding of structural behavior and making feasible decisions. The above mentioned research topic are mainly done to achieve these objectives:

- Detecting the existence of damage
- Finding the location of damage
- Estimating the extent of damage, and

- Predicting the remaining fatigue life.

The practical method to investigate the structural condition is getting some results from dynamic responses after dynamics-based methods. The attractiveness of dynamic responses are due to this fact that we are able to detect and locate damage by them. Dynamic responses are directly related to global behavior of structure and they can provide rapid inspection of large structural systems. The dynamics-based methods can be divided into four groups:

1. Spatial-domain methods
2. Modal-domain methods
3. Time-domain methods, and
4. Frequency- domain methods.

Regarding sensor types and optimum sensor locations, sensor types in structural control and their applications in bridge engineering are summarized as follows:

1. In general, there are kinds of sensors for on-line monitoring, such as piezoelectric transducer, optical fiber sensors, embedded bragg grating sensors, etc. The reliability and durability of these sensors have being testified in many large bridges and tall buildings. Unlike many mechanical systems, typical civil engineering structures are often large in size and therefore have very low natural frequencies. In addition, the vibration level of the structural responses is very often quite low except under strong earthquake. Therefore, the sensors of a monitoring system must be able to work in a very low frequency range and they must have a large dynamic measurement range. The industry has made great achievements in developing sensor and is still working forward [10,13].
2. ISIS CANADA as a pioneer research center in the structural health monitoring field has published a report which includes useful information about sensors and sensing systems. It is a very good reference about sensors [14].
3. A new sensor is introduced, maximum and cumulative displacement memorizing sensor. This sensor is able to measure the maximum, minimum, present and cumulative displacement without any battery. The sensor system consists of linear potentiometer, rotation potentiometer, transmitter and noncontact reader (receiver). This sensor system is suitable for memorizing the deformation of aseismic dampers. This system has installed in several buildings and been verifying the effectiveness [15].

On the other hands, optimum sensor locations for damage detection in bridge engineering are mainly summarized as follows:

1. An effective independence algorithm based on the contribution of each sensor location to the linear independence of the identified modes is proposed. The initial candidate set of sensor locations was quickly reduced to the number of available sensors [16,17].
2. The effective independence method is extended in an algorithm where sensor placement is achieved in terms of the strain energy contribution of the structure [18].
3. A methodology for optimum sensor locations is proposed for parameter identification in dynamic systems [19].
4. A rational statistical-based approach is developed to the optimal location of sensor based on Fisher's information matrix for the model parameters [20].

5. An optimal sensor placement is reported for the purpose of detecting structural damage. The prioritization of sensor locations is based on an eigenvector sensitivity analysis of a finite element model of the structure [21-23].
6. A method is presented in which the sensor locations are prioritized according to their ability to localize structural damage based on the eigenvector sensitivity method. Numerical examples and test results show that this approach is effective for detecting structural damage directly using optimum and incomplete test modes [24].
7. Another new idea is that neural network techniques are used to place sensors. In an attempt the neural network and methods of combinatorial optimization are used to locate and classify faults [25-28].
8. By extensive simulation and experiments for sensor network, it is investigated and verified that the higher the oversampling frequency, the broader the bandwidth and higher signal-to-noise ratio [29-31].

The static and dynamic data are collected from all kinds of sensors which are installed on the measured structures. And these data will be processed and usable information will be extracted. So the sensitivity, accuracy, and locations, etc. of sensors are very important for the damage detections. The more information is obtained, the damage identification will be conducted more easily, but the price should be considered. That's why the sensors are determined in an optimal or near optimal distribution. Then, the theory and validation of optimum sensor locations will still being developed.

3. SAFETY EVALUATION SYSTEM BY IN-SITU LOADING TEST USING LOADING VEHICLE FOR SHORT & MEDIUM SPAN BRIDGES

As one of the useful data collecting systems related to structural safety assessment such as load carrying capacity or durability in existing bridge site, a German research team dealt with an unique experimental safety evaluation system(both loading vehicle and measuring equipment) by in-situ loading tests for mainly short and medium span bridges[4,32]. In order to optimize the assessment procedure, methods and equipment of the system have been established for increasing refinement and complexity from following points of view:

- to adequate the formulation of a German national technical guideline for loading tests [33]; load tests up to service load (e.g. under normal traffic operation) ,
- to make clear the assessment procedure of the actual load carrying capacity may prove directly by using a special designed loading vehicles and special loading technology for mainly short and medium span bridges , and
- to determine the structural safety experimentally by performing an in-situ loading test without causing any damage which would impair the safety or the durability of the target bridge.

3.1 Concept of the Experimental Safety Evaluation by an In-situ Loading Vehicle, BELFA[4]

For purpose of experimental safety evaluation in-situ for a target bridge as a structural health monitoring system, a special designed(mobile) loading vehicle has been developed for performing loading tests at the road bridge site in a more effective way[34]. It is named "BELFA" after the German word "Belastungsfahrzeug" for loading vehicle and may be moved on public roads. It

allows conducting these experiments without the cost and timing consuming construction of reaction frames[www.belfa.eu]. The project was funded by the Bundesministerium für Bildung und Forschung BMBF (about 1.85 million euros). The loading vehicle as the final design, BELFA, consists of a tractor truck with 4 axels and a special semi-trailer with 5 axles as shown in [Figure 2](#). BELFA has 80 tons in total weights, the length amount to 22.5 m in transportation mode and up to 35.5 m in testing mode (see [Table 1](#)). It may be also used to apply loads up to 1500 kN so that the common load levels and loading diagrams can be conducted (e.g. in Germany: bridge class 60, which is the highest bridge class in the German standard DIN 1072). As shown in [Table 1](#), BELFA has generally the technical limitations that a span of target bridges has to be less than 18m, that is short and medium span bridge, and to be smaller than Germany bridge class 60 loads as the maximum applied load in-situ.

As the operation sequence, the loading vehicle, BELFA is driven to the target bridge using its own tractor truck as shown in [Figure 3\(a\)](#). A special driving permission is needed every time since BELFA has over length and overweight. It stops in front of the bridge where the truck is pulling out the front part while the rear axles are blocked. The saddle score of the truck can be reduced additionally by de-airing the pneumatic shock absorption of the last three axles. Thus the truck may cross the bridge, which load carrying capacity is still unknown, with the lowest possible weight. To ensure a riskless crossing it is expedient to assess the structural behavior using the measurement equipment that was installed before. When the first of the rear axles is reaching the bridge, they are blocked again and the middle part is pulled out, too (see [Figure 3\(b\)](#)). BELFA is placed in the right position and fixed in the desired length. The maximum distance between these supports amounts to 18 m which limits the span of the bridges to be tested to this length. The correct position and length is important since the centre of mass must be in accordance with the planned loading positions. Afterwards all axles are de-aired and are fixed to the vehicle chassis in order that all of the masses were usable as counter-force for the test loading. Four hydraulic jacks, two in the front and two in the rear, are lifting up the whole vehicle so that it serves like a reaction frame as shown in [Figure 3\(c\)](#). Now starts the self-driving deck crane, which can be moved in the longitudinal direction, to place entrained weights on the chassis. If this weight is not sufficient for compensating the test loads applied, additional ballast weight is required. For that, an on-board water bag may be filled with up to 20 tons of water. If still the reaction force is too low, the BELFA may be anchored at the bridge supports by using high-strength steel bars or may be ballasted additionally with external weight. Doing so, the centre of mass must be adjusted to the desired position. The hydraulic actors were moved to the test position, which may be at any point between the supports, and their load distribution beams were lowered to the pavement.

During a load test at the bridge site as shown in [Figure 4](#), the existing bridge components such as main girders and slabs with an unknown effective load carrying capacity is assessed. Though the effects of loading on the bridge (e.g. deformations) can be measured, the magnitude of the present dead load as well as the ultimate strength is unknown. All equipment must work electronically and has to be integrated in an online measurement system to ensure that the critical load level is definitely identified, which is characterised by beginning damage processes. The experimental safety evaluation without causing any damage is tied to two important technical prerequisites:

- The application of the test loads has to be undertaken in a way that sudden failure of the structure is avoided even in the case of unexpected damage.
- During test loading, the behaviour of the structure has to be monitored continuously and evaluated in real time (e.g. deformations and micro crack formation). In this way, critical load levels are identified and the loading program may be altered in order to avoid damage to the

structure. Limit criteria have to be determined prior to the test according to the recommendations [33].

The best results concerning a safety evaluation are obtained if the structural behaviour above the service load level can be assessed experimentally.

3.2 Safety Assessment and Application to Actual Bridges[4]

The German guideline for loading tests [33,35] shows the equation how to calculate the applied test load, $ext F_{target}$ for the target bridge,

$$ext F_{target} = \sum_{j>1} \gamma_{g,j} \cdot G_{k,j} + \gamma_{q,1} \cdot Q_{k,1} + \sum_{i>1} \gamma_{q,i} \cdot \psi_{o,i} \cdot Q_{k,i} \quad (1)$$

where, $0,35 G_{k,1} \leq ext F_{target} \leq ext F_{lim}$,

$G_{k,1}$: Characteristic value of existing structural permanent action,

$G_{k,j}$: Characteristic value of additional self-weight j ,

$Q_{k,1}, Q_{k,i}$: Characteristic value of variable action (traffic loads) respectively of main variable action i ,

$\gamma_{g,j}$: Partial safety factor of permanent action G ,

$\gamma_{Q,1}, \gamma_{Q,i}$: Partial safety factor of variable action Q ,

$\psi_{o,i}$: Combination coefficient in case of several variable action Q .

The particular values for partial safety factors and for combination coefficient can be found in pertinent recommendations or national appendices [36]. However one special feature of load tests is that the target bridge exists and the self-weight is already acting. Thus this safety margin must not be included in the calculation of the test target load $ext F_{target}$ ($\gamma_{g,1} = 1.0$). Dynamic effects also are considered by increasing the variable action using a dynamic coefficient, φ which is dependent on the bridge span. The maximum value of φ is 1.4 for short bridges and cannot fall below 1.0 if the span exceeds 50 m [35].

During the loading test, influential factors can reduce the stress level, thus the attempted load conditions in the structure are not reached, e.g.:

- system influences (part restraint, abutment settlement, lateral distribution)
- structural layers (sealing, slope/safety concrete, pavement concrete, road surfaces)
- formation of the cantilevers and caps (incl. solid railings)
- ambient influences (e.g. temperature)

If they do not have the same permanent effect on the structure, the test target load must be increased by an overload ΔF to compensate this reduction [37]. Then the effective applied test load for the target bridge is $ext F_{target} = F + \Delta F$ and generates the stress level in the structure to be verified according to the recommendations. Prior investigations have shown that e.g. pavement acts as a quasi-monolithic strengthening of the structure.

As an example of applying the BERFA in-situ loading system to the target bridge which constructed in 1970 and crossed a river in Germany (BS-bridge) with a span of 12 m length. Its cross section consisted of 19 prestressed precast concrete beams which were completed with poured in-situ

concrete as shown in **Figure 5**. Identical loading tests (place and magnitude) were performed in the following conditions of the bridge structure (**Figure 6**):

- (1) initial condition,
- (2) after removing the asphalt,
- (3) after removing asphalt and protection/sloping concrete,
- (4) after removing asphalt, protection/sloping concrete, pavement concrete.

As a result, **Figure 6** shows the load deflection curves of first loading cycle at midspan under above four conditions. In this case the determined overload factor was 1.04 ($\Delta F = 0.04 \times F$) taking only the influence of asphalt into account. Further studies have shown that the coefficient can rise up to 1.26. In the case of constant factors such as restraint effects from concrete joints or earth resistance on the abutments, it is presumed that this influence has the same favourable effects for the residual use period. These influences do not have to be taken into account in the form of an overload and can be used therefore as potential for a higher experimental bridge rating.

As another example, **Figure 7** shows an existing reinforced concrete short-span bridge ($L = 5.80$ m) which crossed a small stream in a rural area. The bridge investigation assessed several serious damages as cracks, corrosion and spalling. The permitted loads were reduced to a total weight of 2.8 tons, so that the primary use of the state road was brought to an end. Load tests should proof if the permitted loads could be assessed for 16 tons (DIN 1072, [35]) at least.

They lead to a simplification of the testing program. The load tests could be reduced to one half of the bridge. However, this was necessary since the bridge laid in a bend and BELFA could be positioned only on one half of the bridge as shown in **Figure 8**. The experimental safety evaluation involved the superstructure, the supports, the abutments and the foundations. The load was increased infinitely variable using the dead weight of the BELFA as counter force, which was supported as far away from the bridge as possible to avoid interaction neither with the abutments nor the foundation. The target loads were calculated in advance to reach the maximum stress conditions of each building element. They took into account all dead loads and live loads according to the recommendations including partial safety factors. Several reactions were measured during the tests to analyse the particular condition of the structures, e.g. deflections, settlement of abutments, crack width, steel strain. They were recorded and at the same time monitored on a screen as load-reaction-curve.

The successful experiments were concluded as following:

- The deformation behavior (superstructure / foundation) is predominantly linear-elastic
- The deformations are smaller than the calculated (conservative) prognosis
- The load distribution across the bridge is intact – several girders share the load
- Cracks open and close under load, critical crack widths were not reached
- The massive parapets take some of the load
- High wheel loads lead to longitudinal cracks in the slab between the girders (visible, $w \leq 0.1$ mm)

3.3 Summary

In here, a special designed mobile loading vehicle, BELFA for road bridges has been introduced into the technology of in-situ bridge testing which is a structural health monitoring system for existing short and medium span bridges. However, it needs to special attention on the loading process, such

as the actuators position and spacing, cycling numbers, etc. The proposed in-situ loading test procedure can be assessed accurately results of the structural safety based on the actual bridge conditions but has technical or economical limitations. Furthermore, the applicability of the procedure has been proved by successful application to several existing bridges by periodic inspections according to the German guidelines.

4. CONTINUOUS VEHICLE-BASED MONITORING SYSTEM FOR SHORT & MEDIUM SPAN BRIDGES

In order to properly maintain numerous short and medium span bridges in Japan by conducting periodic inspections, it is necessary to develop an easy-to-use and efficient bridge monitoring system[38-40]. The authors are developing a short and medium span bridge monitoring system for detecting deterioration in safety and other performance of existing bridges by use of under-rear-wheel-spring acceleration sensors installed on fixed-route buses (hereinafter referred to as the "bus monitoring system")[41,42]. The proposed monitoring system represents a monitoring system, to be applied mainly to short and medium span bridges, making use of a fixed-route bus operated as part of a public transit system. The aim of the proposed system is to detect the transition from the "acceleration period(stage)," in which the safety performance of a bridge sharply declines because of aging, to the "deterioration period(stage)" (see [Figure 9](#)). This paper presents the results of total four-year field test of the bus monitoring system conducted by using an in-service fixed-route bus in Ube City, Japan. For the bus monitoring system, "characteristic deflection" was defined as an indicator that may be useful in efficiently detecting a structural anomaly of a bridge. For the purpose of examining the sensitivity of characteristic deflection, the effect of artificial damage (bridge guardrail removal) on characteristic deflection was evaluated by using an out-of-service bridge being removed. Another field test was conducted jointly with the Universidade Nova de Lisboa (New University of Lisbon) in Portugal as an overseas application of the bus monitoring system. By analyzing the field test results thus obtained, possible steps in realizing practical application of the bus monitoring system were considered systematically.

4.1 Principles and Overview of the Bus Monitoring System

The bus monitoring system has been developed by the method of monitoring short and medium span bridges by using fixed-route buses operated as part of public transport systems. [Figure 10](#) illustrates the bus monitoring system, and [Figure 11](#) shows the analysis flow of the monitoring method. Main reasons why the use of fixed-route buses, which are large and heavy vehicles, was considered are as follows:

- 1) If a large vehicle about 10 m in length is used for on-the-move measurement on a short or medium span bridge, that vehicle is likely to be the only vehicle passing over the bridge at point in time.
- 2) In order to make a short or medium span bridge, whose stiffness is relatively high, it is necessary to use a reasonably heavyweight vehicle.
- 3) Large vehicles used to vibrate bridges make it easier to reproduce measuring conditions such as travel time, route, frequency of passages and vehicle speed.
- 4) By using a fixed-route bus equipped with acceleration sensors, main short and medium span bridges in the area of interest can be monitored on a regular basis. By so doing, monitoring cost would be substantially less than in the case where sensors were installed at all bridges.

5) Power for the measuring devices can be supplied from the power supply available in the bus.

4.1.1 Theoretical background

This section describes in detail the principle of operation of the bus monitoring system: how bridge anomalies are detected from vehicle vibration as proposed in a preceding study [47].

a) Similarity between bridge response and the under-spring response of the bus

The case in which a vehicle crosses a bridge can be represented by a dynamic interaction between the equation of motion expressed by Eq. (2) and the equation of motion expressed by Eq. (3). Thus, structural models of the bridge and the vehicle are formulated with different equations of motion, and interactions at points of connection between them are expressed by input and output vectors. This approach is called the "substructure method" [48].

$$M_m \ddot{\delta}_m + C_m \dot{\delta}_m + K_m \delta_m = \begin{Bmatrix} F_s \\ F_m \end{Bmatrix} \quad (2)$$

$$M_s \ddot{\delta}_s + C_s \dot{\delta}_s + K_s \begin{Bmatrix} \delta_{sf} \\ \delta_{sg} \end{Bmatrix} = \begin{Bmatrix} 0 \\ F_s \end{Bmatrix} \quad (3)$$

where,

M_m, C_m, K_m : mass/damping/stiffness matrix on the bridge side,

$\ddot{\delta}_m, \dot{\delta}_m, \delta_m$: response acceleration/velocity/displacement matrix on the bridge side,

M_s, C_s, K_s : mass/damping/stiffness matrix on the vehicle side,

$\ddot{\delta}_s, \dot{\delta}_s, \delta_{sf}$: response acceleration/velocity/displacement vector on the vehicle side,

δ_{sg} : input forced displacement vector on the vehicle side,

F_m, F_s : support reaction vector on the vehicle side.

To express the interaction between the bridge and the vehicle, the under-spring reaction of the vehicle is input to the bridge side as load vector, F_s , and bridge deflection ($\delta(t) \in \delta_m$) and road surface roughness, $\lambda(t)$ are input as forced displacement vector, δ_{sg} . The bridge–vehicle system at time, t to $t + \Delta t$ when the vehicle passes the bridge can be simply represented by a three-mass–interaction spring–mass model as shown in **Figure 12**. The vibration of this system is caused by the vehicle vibration induced by the input of the road surface roughness ($\lambda(t)$) and the bridge deflection, $\delta(t)$, and the excitation to the bridge due to the reaction.

The first step is to consider the case where various physical parameters of the bridge system and the vehicle system and the road surface roughness, λ remain constant during a certain measurement period. Naturally, the same measurement results are obtained every time from this interaction system.

The next step is to consider the case where various physical parameters for the vehicle system and the road surface roughness, λ remain constant and the stiffness, K_m of the bridge has changed because of some kind of damage. In this case, the measured value of bridge deflection, $\delta(t)$ due to

the reaction from the vehicle system at a given time t also changes. As $\delta(t)$ changes, vehicle system nodal response, $\ddot{\delta}_s, \dot{\delta}_s, \delta_{s_f}$ also changes. Furthermore, as the vibration of the vehicle system changes, vehicle system reaction, that is, exciting force, F_s changes so that the bridge deflection, $\delta(t + \Delta t)$ changes. As a result of this chain of changes, effects of the change in the stiffness, K_m of the bridge appear in the measurement results obtained from both the bridge system and the vehicle system. Thus, structural anomalies of the bridge due to deterioration, etc. emerge as changes in vehicle system nodal response, $\ddot{\delta}_s, \dot{\delta}_s, \delta_{s_f}$. It is therefore possible, in theory, to detect bridge anomalies from the vehicle side.

In the case of the proposed system, detection becomes easier as $\delta(t)$ increases. This is why large (heavy) vehicles are more suitable for monitoring than smaller vehicles. According to measurement data [49,50] in the case of a large vehicle, M_A tends to be greater than M_B ($M_A > M_B$), and K_s tends to be smaller (light) than K_t ($K_s < K_t$). This means that the under-spring part (Node B) of the vehicle is more sensitive to changes in the condition of the bridge than the over-spring part (Node A) as shown in [Figure 13](#). For the purposes of this study, therefore, it was decided to pay attention to under-spring vibration. It was also decided to measure acceleration in order to realize a relatively simple vibration measurement system. If bridge vibration is to be estimated from the under-spring vibration of the bus, it is necessary to determine how they are correlated.

Let us consider the upper body/lower body/bridge substructuring scheme as shown in [Figure 14](#). The equation of motion for a given system is given in the form of a second-order differential equation:

$$M\ddot{\delta}(t) + C\dot{\delta}(t) + K\delta(t) = F(t) \quad (4)$$

where, M , C and K are lumped mass, damping and stiffness matrices for a given system; and $\delta(t)$, $\dot{\delta}(t)$, $\ddot{\delta}(t)$ and $F(t)$ are displacement, velocity, acceleration response and external force vectors, respectively, for a given system at time t .

Let differential operator D and shift operator Z be expressed as,

$$D \cdot \delta(t) = \frac{d\delta(t)}{dt} \quad (5)$$

$$Z \cdot \delta(t) = \delta(t + \Delta t) \quad (6)$$

Then, the equation of motion in Eq. (4) can be rewritten as,

$$M \cdot D^2 \delta(t) + C \cdot D \delta(t) + K \delta(t) = F(t) \quad (7)$$

Taylor expansion of Eq. (6) gives,

$$\delta(t + \Delta t) = \delta(t) + \Delta t \frac{d\delta(t)}{dt} + \frac{\Delta t^2}{2!} \frac{d^2\delta(t)}{dt^2} + \frac{\Delta t^3}{3!} \frac{d^3\delta(t)}{dt^3} + \dots + \frac{\Delta t^i}{i!} \frac{d^i\delta(t)}{dt^i} + \dots \quad (8)$$

Using Eq. (5), we can obtain

$$\delta(t + \Delta t) = \left\{ 1 + \Delta t D + \frac{\Delta t^2 D^2}{2!} + \frac{\Delta t^3 D^3}{3!} + \dots + \frac{\Delta t^i D^i}{i!} + \dots \right\} \delta(t) \quad (9)$$

If Newmark's β method ($\beta = 1/4$) is used, the differential operator relation can be assumed as follows:

$$D\Delta t \doteq \frac{2(1-Z^{-1})}{1+Z^{-1}} \quad (10)$$

Substituting this in Eq. (7) gives Eq. (11):

$$M \cdot \left(\frac{\Delta t}{2} \cdot \frac{(1-Z^{-1})}{1+Z^{-1}} \right)^2 \delta(t) + C \cdot \left(\frac{\Delta t}{2} \cdot \frac{(1-Z^{-1})}{1+Z^{-1}} \right) \delta(t) + K\delta(t) = F(t) \quad (11)$$

Let k represent a post-discretization step at time t , and $k+1$, the next step. Then, Eq. (6) can be rewritten as $Z \cdot x(k) = x(k+1)$. Hence, Eq. (11) can be reduced to the difference equation:

$$\begin{aligned} \left\{ M + \frac{\Delta t}{2} C + \left(\frac{\Delta t}{2} \right)^2 K \right\} \delta(k) - \left\{ M - \left(\frac{\Delta t}{2} \right)^2 K \right\} \delta(k-1) + \left\{ M - \frac{\Delta t}{2} C + \left(\frac{\Delta t}{2} \right)^2 K \right\} \delta(k-2) \\ = \left(\frac{\Delta t}{2} \right)^2 \{ F(k) + 2F(k-1) + F(k-2) \} \end{aligned} \quad (12)$$

The right-hand side and the second and third terms of the left-hand side of Eq. (12) are known when solving the equation at step k (= time t). Let $F'(t)$ represent the right-hand side of the equation; $C_0(M, C, K, t)$, the second and third terms of the left-hand side; and $P(M, C, K, t)$, the coefficient of $\delta(k)$ of the first term of the left-hand side. Then, Eq. (12) can be rewritten as,

$$\delta(t) \cdot P(M, C, K, t) + C_0(M, C, K, t) = F'(t) \quad (13)$$

where, $P(M, C, K, t)$ is a proportionality coefficient dependent on the system at time t . On the assumption that difference approximation is valid, the equation of motion for the wheel-bridge system can be written, by using the proportionality coefficient P and the state constant, (known value), as shown in Eq. (13) [51]. This means that the response to the input vector is distributed proportionately depending on system-dependent constants.

Next, let us consider the vibration of the wheel-bridge system due to the force transmitted from the upper part of the vehicle.

As in the case mentioned earlier, the response of the wheel-bridge system to the input from the upper part of the vehicle is distributed proportionately depending on physical constants of the system. It can be inferred, therefore, that if A_b represents the bridge response vector and A_s represents the bus wheel response vector, then matrix P that satisfies the following equation under continuously changing conditions:

$$A_b = P^{-1} A_s P \quad (14)$$

b) Extracting damage and deterioration related information from under-spring vibration of the bus

This section describes the concept of the method of extracting damage and deterioration related information from the vertical under-spring vibration of the bus without being affected by the dynamic characteristics of the bridge and the vehicle and road surface roughness. The vertical under-spring vibration response, $\delta_a(t)$ of a bus traveling at a constant speed can be expressed as the sum of static displacement, $\delta_{sa}(t)$, which is dependent on the stiffness of the bridge and the weight of the bus, and dynamic displacement, $\delta_{da}(t)$, which is dependent on road surface roughness and the vibration characteristics of the bridge and the vehicle:

$$\delta_a(t) = \delta_{sa}(t) + \delta_{da}(t) \quad (15)$$

If road surface roughness is assumed to be a stationary random Gaussian process with a mean value of 0 and if dynamic displacement including the bridge–vehicle interaction is assumed to be an ergodic process and therefore Fourier-expandable, the dynamic displacement, $\delta_{da}(t)$ can be expressed as their sum:

$$\delta_{da}(t) = S_r(\Omega, t) + \int_{-\infty}^{+\infty} X(f) \cdot e^{2\pi f t} df \quad (16)$$

where, $S_r(\Omega, t)$ is a density function for road surface roughness; Ω , the spatial frequency of the road surface; and $X(f)$, a Fourier series. In Eq. (16), the limit of the sample average of the second term is 0. The mean value, therefore, of N samples, where N is a sufficiently large number, obtained from $\delta_{da}(t)$ can be expressed as,

$$\frac{\sum_{t=1}^N \delta_{da}(t)}{N} \doteq 0 \quad (17)$$

As the next step, a total of k samples are taken from measured values of $\delta_a(t)$, and their mean value can be expressed, by representing their mean by $\overline{\delta_a(t)}$, as,

$$\overline{\delta_a(t)} = \frac{\sum_{j=1}^k \delta_a(j)}{k} \quad (18)$$

Since the distribution of sample means should be normal according to the central limit theorem, $\overline{\delta_a(t)}$ should converge to a certain value, μ_a . For a sufficient number (n) of sample means $\delta_a(j)$, therefore, it can be expressed as,

$$\mu_a = \frac{\sum_{i=1}^n \overline{\delta_a(t)}_i}{n} = \frac{\sum_{i=1}^{nk} (\delta_a(j))_i}{nk} \quad (19)$$

If sampling from $\delta_a(t)$ is done so as to avoid duplication and N is sufficiently large relative to $nk = N$ and $t = 1$ to N , then the following approximation can be made:

$$\mu_a \doteq \frac{\sum_{t=1}^N \delta_a(j)_t}{N} \quad (20)$$

This expresses the average vertical under-spring displacement of a bus crossing a bridge. This can be rewritten, on the basis of Eqs. (15) and (17), as,

$$\mu_a \doteq \frac{\sum_{t=1}^N \delta_{sa}(t)}{N} \quad (21)$$

This means that the average of sample values obtainable from a sufficiently large number (N) of measured values of vertical under-spring displacement of a bus crossing a bridge can be extracted as values (μ_a) that are relatively unaffected by the vibration characteristics of the bridge and the vehicle and the dynamic displacement due to road surface roughness. The μ_a thus obtained is referred to as "characteristic deflection".

Means of deflection, $\delta_b(t)$ at a given point on the bridge when a vehicle crosses it also converge to a certain value, μ_b , relatively unaffected by dynamic deflection by making similar assumptions. In a similar way, μ_b can be expressed, by using static deflection, $\delta_{sb}(t)$, as,

$$\mu_b \doteq \frac{\sum_{t=1}^N \delta_{sb}(t)}{N} \quad (22)$$

If the law of similarity mentioned earlier holds true with respect to $\delta_{sa}(t)$ and $\delta_{sb}(t)$, they can be related as,

$$\mu_b = P\mu_a \quad (23)$$

Let μ_a and μ_b represent values extracted from values obtained from measurement when the bridge is in a sound condition and μ'_a and μ'_b represent values extracted from values obtained after the occurrence of deterioration or damage. Then, the change ratio, α expressed as,

$$\alpha = \frac{\mu'_b}{\mu_b} = \frac{\mu'_a}{\mu_a} \quad (24)$$

And it can be defined as a parameter for structural anomaly detection. After setting the value of α , "characteristic deflection" is monitored, and if it has exceeded a certain limit, it can be deemed to indicate that the latter half of the damage acceleration period of the bridge has ended and the bridge has entered the deterioration period. In reality, however, bus operation is affected by not only the static displacement, $\delta_{sa}(t)$ and the dynamic displacement, $\delta_{da}(t)$ expressed by Eq. (15) but also external disturbance factors, $\delta_x(t)$, such as weather and oncoming vehicles. For the purposes of this study, the vertical displacement, $\delta_a(t)$ including the influence of external disturbance factors, $\delta_x(t)$ is expressed as

$$\delta_a(t) = \delta_{sa}(t) + \delta_{da}(t) + \delta_x(t) = \delta_{sa}(t) + S_r(\Omega, t) + \int_{-\infty}^{+\infty} X(f) \cdot e^{2\pi f t i} df + \delta_x(t) \quad (25)$$

and its characteristics and actions to be taken are considered by using a real in-service fixed-route bus.

4.1.2 Characteristic deflection calculation method

For the purposes of this study, characteristic deflection, μ_a [41,43] is defined as the estimated deflection obtained from Eq. (25) averaged in the bridge section of the road. Studies have shown

that characteristic deflection can be made to converge to the extent that anomaly evaluation results are not affected by increasing the number of samples, N , by using the central limit theorem, in order to allow for the influence of various external disturbance factors, $\delta_x(t)$, including the operating conditions of the bus. In this study, the authors used the moving average method to process characteristic deflection data and see what happens. In the moving average method, averages are calculated for incrementally shifted data sections. **Figure 15** shows the flow of “characteristic deflection” calculation after the acquisition of measurement data from the acceleration sensor installed to the bus. Each step is described below in detail. The step numbers ("Step 1" to "Step 5") shown below correspond to the numbers shown in **Figure 15**.

Step 1: Extract data on vertical acceleration during bridge crossing

Data on vertical acceleration during bridge crossing are extracted from acceleration sensor measurement data by referring to a combination of other data such as the time at which the buses crossed each bridge and GPS data.

Step 2: Estimate the time at which the midspan point was passed

Extracted data on acceleration during bridge crossing include considerable vibrations recorded at joints. It is therefore necessary to use midspan acceleration data that do not include such joint vibrations. The time at which the midspan point of a girder was passed can be estimated by identifying bridge sections meeting such criteria as duration and wave count and extracting relevant data from non-joint data. It may be difficult, however, to identify joint locations because acceleration data may vary in magnitude depending on such factors as bus operating conditions. For accurate estimation of the time at which the midspan point was passed, therefore, attention is paid to estimated deflection diagrams obtained by integrating vertical acceleration data twice. As an example, **Figure 16** shows acceleration waveforms measured recently and estimated deflection waveforms obtained by integrating the bridge-crossing acceleration data twice. As shown in **Figure 16**, characteristic waveforms appear when the bus crosses a bridge. The waveforms at midspan, therefore, are estimated and identified by synchronizing different data on the basis of the characteristic waveforms appearing in the estimated deflection diagram when the bus passes the joints of the bridge of interest.

Step 3: Extract data on vertical acceleration during bridge crossing

Extract the midspan vertical acceleration data identified at Step 2. **Figure 17** shows examples of joint and midspan portions of vertical acceleration data. In the example shown in **Figure 17**, a portion of midspan region data corresponding to about 0.8 sec (about 10 waves) was identified as data to be extracted. The most important thing in “characteristic deflection” calculation is to determine the extraction range according to such details as wave count and duration and extract acceleration waveform data from the same segment in every time. Step 3 is described in detail in the next section.

Step 4: Integrate the extracted acceleration data twice

The extracted acceleration data is converted to velocity data by integrating once and to displacement data by integrating twice. In this study, the vertical displacement obtained by integrating the vertical acceleration twice is regarded as estimated midspan deflection during bridge crossing. **Figure 18** shows an example of estimated deflection in this way. As shown in **Figure 18**, the vertical displacement (estimated deflection) at time 0 (shown with a red circle) was assumed to be the initial value of 0 in the selected midspan section.

Step 5: Average estimated deflections

The estimated midspan deflections during bridge crossing shown in the graph are time-averaged to calculate the “characteristic deflection” (see **Figure 18**).

Described above is the procedure for calculating the “characteristic deflection” used as an indicator in the proposed evaluation method. “Characteristic deflections” thus calculated include the effects of external disturbance factors as shown in Eq. (25). It has been confirmed that „characteristic deflections“ calculated as described above are significantly affected by human errors (individual errors). Efforts need to be made, therefore, to minimize human error in the calculation process.

4.2 Long-term Monitoring of Short and Medium Span Bridges along Bus Routes in Ube City

This chapter describes on the long-term field test of the bus monitoring system for short and medium span bridges located on the municipal bus routes in the city of Ube, Yamaguchi Prefecture, Japan conducted over a period of about four years from December 2010 to September 2014. Since the field test has been conducted for about four years, a considerable amount of measurement data has been accumulated. The data thus accumulated were utilized to evaluate the influence of fixed-bus operating conditions (external disturbance factors such as weather, the number of oncoming vehicles, the number of persons in the vehicle and vehicle speed) on characteristic deflection. In addition to the derivation of conversion (correction) factors based on the correlations between various bus operating conditions (external disturbance factors) and characteristic deflection carried out in previous studies [52,53] based on data accumulated over a period of about one year, newly obtained measurement data were used. Thus, by conducting an integrated study using all data accumulated over a combined period of about four years, new study results have been obtained. These results are also described in this chapter.

4.2.1 Overview of long-term monitoring conducted on Ube-city's bus routes

In order to develop and put to practical use a bus monitoring system for short and medium span bridges located on bus routes, it is necessary to conduct a series of studies involving a long-term field test using an in-service fixed-route bus. In this study, with the cooperation of Ube-city's Transportation Bureau(UTB), long-term monitoring of short and medium span bridges located on the city's in-service bus routes has been continued. The study focuses mainly on the following:

- (1) The number of short and medium span bridges existing on the city's bus routes and the total number of existing bridges in need of maintenance.
- (2) The method of calculating "characteristic deflection," which is an indicator of the structural health of bridges based on long-term measurement data and its usefulness in damage detection.
- (3) Proposing a method for long-term observation of characteristic deflection and enhancing damage detection sensitivity.
- (4) Verifying the damage detection sensitivity of characteristic deflection by use of artificial damage and setting "critical characteristic deflection" (criterion value) by use of an analysis model.
- (5) Evaluation of the influence of bus operating conditions (external disturbance factors) on characteristic deflection based on long-term measurement data and an attempt at deriving conversion (correction) factors.

4.2.2 Number of bridges on municipal bus routes and bridges to be monitored

Figure 19 shows the number of bridges under the jurisdiction of Ube-city located on the municipal bus routes [54,55] operated by Ube-city's Transportation Bureau. Of a total of 435 bridges 2 m or more in length managed by Ube-city, 35 bridges are located on the bus routes. Although they account for less than 10% of all bridges in the city, after the bus monitoring system goes into service, all existing bridges in need of maintenance will be monitored with the cooperation of Ube-city's Construction Department. Prior to the long-term field test using the bus monitoring system, short

and medium span bridges to be monitored in the test were selected. **Figure 20** shows the present and future of the bridges managed by Ube-city's Construction Department, comparing the percentages of bridges older than 50 years. As shown, the number of bridges older than 50 years as of fiscal year 2011 is 65, which is about 15% of all bridges. In 2031 (20 years later), it will increase to 323 bridges (about 74% of all bridges), indicating a rapid deterioration of the bridges in the city. Of such short and medium span bridges located on the bus routes in Ube-city, three bridges that are thought to have deteriorated considerably, namely, “Shiratsuchi Daini Bridge (two-span reinforced concrete T-girder bridge)”, “Jase Bridge (five-span prestressed concrete slab bridge)” and “Shingondai Bridge (single-span prestressed concrete girder bridge built by the Bi-Prestressing Method)”, were selected for the long-term monitoring. Specifications and general views of these three bridges are shown in **Table 2** and **Figure 21**, respectively.

4.2.3 Overview of the vehicle used for long-term measurement

This section briefly describes the bus (vehicle) used for the long-term field test. The long-term measurement using Ube-city's municipal bus routes has been continued by using an in-service fixed-route bus (i.e. a bus actually used to transport passengers) owned by Ube-city's Transportation Bureau. By using the three-axis acceleration sensor installed under the rear wheel spring of this vehicle, the vibration properties of the bridge being crossed by the bus were extracted as the acceleration response during bridge crossing and deflection was estimated (Miyamoto and Yabe 2012). **Tables 3 and 4** show the specifications [56] of the bus (vehicle) and the three-axis acceleration sensor installed to the bus, respectively.

Figure 22 shows a general view of the bus (vehicle) used for the long-term monitoring. **Figure 23(a)** shows the acceleration sensor installed under the rear wheel spring of the vehicle. The acceleration sensor was bonded to the underside of the rear wheel spring and was coated with waterproof epoxy resin to protect the sensor over a long period of time. The three-axis acceleration sensor used was positioned so that its X, Y and Z axes were aligned with the direction of travel of the bus, the direction perpendicular to the direction of travel and the vertical direction, respectively. Analog vibration data obtained from the three-axis acceleration sensor in the form of acceleration response were converted to digital data via a data logger and saved in the computer as Excel file data. As shown in **Figure 23(b)**, the cable from the three-axis acceleration sensor was routed through the drain hole in the vehicle floor to the data logger.

4.2.4 Measuring method

Figure 24 illustrates the measuring method adopted for the bus monitoring system. In the long-term monitoring that has been conducted on the bus routes in Ube-city, attempts were made to systematically evaluate the influence of bus operating conditions (weather, the number of oncoming vehicles, the number of persons on the vehicle and vehicle speed), besides the acceleration response recorded with the three-axis acceleration sensor, on characteristic deflection and elucidate and quantify their correlations. During the data measurement, a two-person measuring team rode on the bus. One of them, who sat on a front seat near the bus driver, recorded details such as vehicle speed, the number of oncoming vehicles (if any) and weather conditions. The other person, who sat on a rear end seat, operated and checked on the measuring equipment and recorded the number of persons on the bus and the time at which the bus crossed the bridge in time series while collecting other information on possible external disturbance factors. **Figure 25** shows the types of equipment and devices used and how they looked. As shown in **Figure 25**, by connecting the three-axis acceleration sensor with the measuring and display equipment, the person at the rear end seat was enabled to monitor vibration waveforms in real time. The plan for the operation of the bus

monitoring system in the coming years assumes the use of the power supply of the bus for the acceleration sensor, data logger and the computer. In the measurement reported in this study, however, a portable battery was used (see [Figure 25](#)).

4.2.5 Experiment results

Before reporting the results of the long-term field test conducted over a period of about four years, this section touches on some fundamental findings from previous studies. First, a study was conducted to determine whether it is possible to extract the "estimated deflection" (basic data for the calculation of "characteristic deflection") of the bridge of interest from the rear wheel under-spring acceleration response of a bus (vehicle). In that study, an acceleration sensor was installed in the midspan zone of the Shingondai Bridge (prestressed concrete girder bridge built by the Bi-Prestressing Method), which is one of the three bridges selected for the present study, and the acceleration response of the bridge and the under-spring acceleration response of the bus were compared in time series. Next, another study was conducted to evaluate the influence of bus operating conditions during bridge crossing on characteristic deflection and use the findings for conversion (correction) factor derivation in future. In the study, coefficients of correlation between those conditions and characteristic deflection were derived. Although the goal of conversion (correction) by use of correlation coefficients was not achieved because the required amount of data was not available, the study succeeded in showing that the variability of characteristic deflection can be reduced by applying the moving average method to a time series. A vehicle-induced vibration simulation taking account of the coupling with the bus and the bridge, etc. was also performed by using the substructure method [48], which is a technique classified as a finite element method (FEM). The aim of the simulation was to develop "serious deterioration (damage) criteria" by which to determine the degree of increase in "characteristic deflection" that can be deemed to be the onset of serious damage leading to the transition to the deterioration phase of a bridge.

a) Time series comparison of rear wheel under-spring acceleration response of the bus and midspan acceleration response of the bridge

As a basic check, it is necessary to determine whether it is possible to detect damage from the under-rear-wheel-spring acceleration response of the bus when a serious structural anomaly of a bridge has occurred. In other words, it is necessary to check whether the under-rear-wheel-spring response and the bridge response are coupled. This section looks at the correlation in terms of vibration properties during bridge crossing by using data obtained from another acceleration sensor installed to the "Shingondai Bridge" mentioned earlier.

[Figure 26](#) shows an example of the relationship between the acceleration sensor location and the path of the bus. The conditions under which the bus actually crossed the bridge were as follows:

Weather: rain

Vehicle speed: 35 km/h

Number of oncoming vehicles: 1

Total number of persons on the bus (including the bus driver): 10

[Figure 27](#) shows an example of an acceleration waveform recorded when the bus was in the midspan zone of the bridge. As shown, the movement of the bus is coupled with the bridge vibration. The two acceleration response waveforms thus obtained from the measurement were analyzed by applying FSWT (Frequency Slice Wavelet Transform) [44,57], which is a time–frequency space

analysis technique, to determine whether there is time-series similarity in vibration properties between the vehicle and the bridge.

Figure 28(a) compares the acceleration response waveforms of the bus and the bridge recorded when the bus passed the midspan zone of the bridge. Figure 28(b) summarizes the FSWT analysis results. As shown, the response waveforms show similarity except at higher-order frequencies corresponding to external disturbances although the vertical axis (acceleration) of the acceleration response waveform needs to be adjusted by reducing the under-rear-wheel-spring response (see Figure 28(a)). As can be seen from the FSWT analysis results shown in Figure 28(b), the bridge underwent coupled vibration at around 12 Hz [44,57] when the bus crossed the bridge. It has also been confirmed through measurement that the wheels under the springs of the bus were vibrating at around 12 Hz regardless of crossing the bridge or not [45]. From this, it can be concluded that the vibration properties of the bridge can be identified from the under-rear-wheel-spring vibration of the bus by using the similarity between them.

b) Proposed method of characteristic deflection monitoring by use of the moving average method

“Characteristic deflection“ is affected by various external disturbances such as the bus operating conditions mentioned earlier. Consequently, „characteristic deflection“ is inevitably subject to variation. An attempt was made, therefore, to determine changes over time in „characteristic deflection“ obtained from the bus monitoring system by applying the moving average method, assuming that as the number of samples, N , increases, variations due to external disturbances such as bus operating conditions converge to a single value according to the central limit theorem. The moving average method is the method of calculating the average of data in data section (segment; the number of data sets to be averaged) by calculating averages for incrementally shifted subsections. In the previous studies, the simple moving average method, which is one of the commonly used moving average methods, was used to process characteristic deflection data. As an example, Figure 29 shows the relationship between the number of data sections (segments) and the standard deviations of the corresponding „characteristic deflections“ obtained by applying the moving average method to data subsets in the data section (segment). As shown in Figure 29, as the number of data sets increases, the standard deviation becomes incrementally smaller. After the number of data sections reaches a certain level, the standard deviation does not change significantly and converges. This is thought to have shown that various external disturbances (error factors) can be characterized by standard deviations and averages if about 15 data sections are used, indicating that the central limit theorem mentioned earlier holds true. In fact, in the bus monitoring system, the standard deviation of characteristic deflection may be deemed to converge if the number of data sections is around 14 or 15. It was therefore decided to use 15 data sections in monitoring time-dependent changes in “characteristic deflection“. On the basis of the concept described above, Figure 30 shows how characteristic deflection (measured value) and the simple moving average change. As shown in Figure 30, measured values of “characteristic deflection“ vary considerably, while simple moving averages of „characteristic deflection“ are noticeably better in terms of variability.

c) Development of serious deterioration (damage) criteria

In the previous studies, characteristic deflection corresponding to the state of bridge damage determined in a vehicle-induced vibration simulation performed by the substructure method, a finite element method, was calculated. The intent was to develop serious deterioration (damage) criteria by which to identify the degree of change (increase) in characteristic deflection that indicates the

occurrence of serious deterioration (damage) of the bridge of interest. In this study, in view of the fact that the bridge under consideration is a prestressed concrete girder bridge (“Shingondai Bridge”, a single-span bridge built by the Bi-Prestressing Method) as mentioned earlier, attention is paid to the decrease in prestressing force as a kind of bridge damage. The National Institute for Land and Infrastructure Management (NILIM) of the Ministry of Land, Infrastructure, Transport and Tourism (MLIT) conducted a study on the relationship between the amount of prestress introduced and displacement (deflection) [46]. On the basis of that study, it has been shown through finite element analysis [47] that if a bridge is damaged so that the amount of prestressing force in a sound condition (100%) decreases by 50%, characteristic deflection increases by a factor of 1.93. It has also been shown that if the bridge is damaged so that the amount of prestressing force decreases by 90%, characteristic deflection increases by a factor of 2.86 [47]. **Table 5** summarizes the relationships of damage representations (sound, Deterioration Phase 1, Deterioration Phase 2) applicable to the “Shingondai Bridge” with the amount of decrease in prestressing force, the equivalent second moment of area and the amount of change in characteristic deflection. The calculated values obtained from the analysis of the seriously damaged bridge of interest (Shingondai Bridge) as mentioned earlier were used as serious deterioration (damage) criteria and compared with the measured changes (2010 to 2013) in characteristic deflection of the Shingondai Bridge. Examples of such comparisons are shown in **Figure 31**. For the purpose of comparison, the characteristic deflection obtained by multiplying the average of the first 15 measurements by 1.93 was used as the serious deterioration (damage) criterion (red line) for Deterioration Phase 1, and the characteristic deflection similarly obtained by multiplying by 2.86 was used as the serious deterioration (damage) criterion (green line) for Deterioration Phase 2. Thus, characteristic deflection is measured continually over a long period of time, and when one of those criterion values is reached, that is deemed to indicate the occurrence of some kind of serious damage in the bridge of interest, and a warning is issued so that necessary actions such as detailed inspection can be taken immediately. Actions such as detailed inspection need to be taken immediately when “characteristic deflection” has reached a criterion level on the out-bound or in-bound or in-bound route of the bus. As of this writing, the observation of the “characteristic deflection” of the “Shingondai Bridge” is underway while comparing the amount of decrease in prestressing force with the serious deterioration (damage) criteria. This approach, however, is not applicable to bridges of other types. For those bridges, it is necessary to identify a number of types of serious damage, taking account of such factors as the characteristics and material used of each bridge, and set serious damage criteria accordingly.

4.2.6 Field-test findings based on four-year monitoring data and discussion

On the basis of the basic findings of the previous studies mentioned in the preceding section, more measurement data have been accumulated (big data) by using the in-service municipal bus network of Ube-city, Japan. This section presents comprehensive verification results based on the monitoring data thus accumulated over a period of about four years. For that purpose, the influence of bus operating conditions (external disturbances) such as vehicle speed and the number of oncoming vehicles on changes in „characteristic deflection“ (indicator used for damage detection) induced by in-service fixed-route bus operation is determined, and their correlations are reflected in conversion (correction) factors. All data on the three bridges that have been monitored by the „characteristic deflection“ observation method proposed in the previous study accumulated over the four years are put together and examined to evaluate the usefulness of the proposed approach in detecting serious deterioration (damage) of the bridges being monitored.

a) Correlations between bus operating conditions and characteristic deflection

The bus monitoring system utilizes an in-service fixed-route bus. Its operating conditions, therefore, act as external disturbances during the long-term observation of „characteristic deflection“. Because of this, as mentioned in Section 4.2.5 (b), the amount of data accumulated in connection with the previous studies was not large enough to identify clear correlations between the bus operating conditions and “characteristic deflection“. “Characteristic deflection“ also varied considerably depending on the bridge concerned and the direction of vehicle movement. Consequently, the results obtained did not make it possible to reflect their correlations in conversion (correction) factors. An attempt is being made, therefore, to reduce the influence of external disturbance factors such as bus operating conditions by applying the simple moving average method. In this section, however, with the aim of evaluating the possibility of insufficiency of accumulated data, correlations between “characteristic deflection“ and the bus operating conditions (external disturbances) are re-examined and discussed by putting together the four-year monitoring data again.

In this study, the coefficients of correlation between the “characteristic deflections“ of the three bridges listed in Table 2 and the bus operating conditions (external disturbances) were calculated. The bus operating conditions that were taken into consideration as possible external disturbance factors included atmospheric temperature (Japan Meteorological Agency data) in addition to the factors considered in the previous studies, namely, weather (clear, rainy), the number of oncoming vehicles, the number of persons on the vehicle and vehicle speed. The data used for calculation were obtained by filtering the collected data according to certain criteria. Table 6 shows the calculation conditions (data filtering criteria) for each bridge. Tables 7(a) and (b) summarize the correlations between the characteristic deflection of each bridge and the bus operating conditions calculated on the basis of the numbers of measurement data sets shown in Table 6. The correlation coefficient ranges and descriptions (definitions) of the correlations shown in Table 6 are as shown in Table 8.

As shown in Table 7, there is no bus operating condition that is strongly correlated with characteristic deflection, showing a coefficient of correlation exceeding 0.7. Turning attention to individual operating conditions, we notice that they show a positive correlation in some cases and a negative correlation in others, and it is difficult to conclude that any of those conditions shows a certain tendency. When calculating characteristic deflection, therefore, it is not possible to reflect correlations with those operating conditions in conversion (correction) factors by using the existing data alone.

In view of these results, it can be concluded that although it can be shown that each of the bus operating conditions (external disturbances) somehow influences the characteristic deflection, at present it is still not possible to quantify such influence so that it can be reflected in conversion (correction) factors. Therefore, as a method of handling variations due to various external disturbance factors including bus operating conditions, the moving average method (simple moving average method) mentioned in Section 4.2.5 (b) was applied for the purpose of observing changes over time.

b) Results of observation of characteristic deflection

On the basis of the study results described in the preceding sections, this section deals with the calculated values of “characteristic deflection“ based on the long-term monitoring of the three bridges in Ube-city's municipal bus network continued over a period of about four years, and the results of observation of changes over time in the characteristic deflection. Table 9 summarizes the measurement data, including the number of data sets, for the three bridges. Table 10 shows the span-by-span averages and standard deviations of „characteristic deflection“ and other related data for the three bridges obtained by processing the four-year data in an integrated manner.

As shown in [Table 10](#), characteristic deflection varies depending on the type of bridge, the direction of vehicle movement and span length. This is thought to be because of the bridge shapes in plan are asymmetric (e.g. curved bridge, skewed bridge, sidewalk on one side). As mentioned in Chapter 2, however, characteristic deflection is a quantity calculated by averaging estimated deflections in the same regions in each bridge or span. The differences mentioned above, therefore, do not pose any problem.

[Figures 32 and 33](#) show changes over time in the characteristic deflection of each span of the two bridges as an example, calculated by applying the simple moving average method mentioned earlier to the four-year monitoring data. As shown, characteristic deflection does not change sharply although it differs somewhat from span to span in the three bridges. It can therefore be concluded that at present the three bridges have not yet undergone serious deterioration (damage). Since, however, the deterioration (damage) of a short and medium span bridge tends to progress rapidly during the acceleration phase, it is necessary to continue long-term observation of changes in characteristic deflection.

c) Discussions

Thinking of bus operating conditions that may affect "characteristic deflection," which is an indicator of the structural health of bridges, as external disturbance factors, the authors tried to quantify the correlations between the bus operating conditions and the characteristic deflection by adding new measurement data to the available data. Although certain degrees of influence of external disturbance factors (bus operating conditions) can be seen, it is as yet not possible to quantify such influence in the absence of a clear tendency or a strong correlation. As a result, it was concluded that at present it is not possible to reflect their correlations in conversion (correction) factors applicable to the bus operating conditions. It was therefore thought that the simple moving average method mentioned in Section 4.2.5 (b) would be useful in treating the influences of the external disturbance factors on the characteristic deflection as variances.

4.2.7 Effect of artificial damage on characteristic deflection

This section reports on the result of a study on the influence of artificial damage (bridge guardrail removal) conducted, by using an out-of-service bridge being removed, to investigate the sensitivity of the bus monitoring system to characteristic deflection.

In the experiment conducted for the purpose mentioned above, acceleration sensors were installed to a sightseeing bus weighing about 15 tons as shown in [Figure 34](#) by using a bridge being removed for the construction of a new bridge. The bridge is a 168.29-meter-long, eight-span reinforced concrete(RC) cantilever T-girder bridge as shown in [Figure 35](#). In consideration of such factors as seasonal changes and the presence or absence of bridge railings, the measurement was conducted four times, namely, in September, 2012, and January, February and March, 2013. [Figure 36](#) shows how the bridge looks before and after guardrail removal.

[Figure 37](#) shows the average values of 15 sets of characteristic deflection measurements calculated from the total of 4 sets of measurements. As can be seen from [Figure 37](#), the concrete guardrails were removed between the second measurement and the third measurement. The characteristic deflection values before and after the guardrail removal, therefore, were compared. The amount of change in the flexural stiffness of the bridge structure due to the guardrail removal was a decrease of about 5%. Examination of Span 3 (see [Figure 35](#)) reveals that the two measured values of characteristic deflection after the guardrail removal (-2.97 , -2.89) were about 10% smaller than the values of characteristic deflection after the guardrail removal (-2.67 , -2.61). This is thought to be because the decrease in the flexural stiffness of the bridge structure influenced characteristic

deflection. However, examination of Span 2, which includes the cantilever structure, reveals that the measured value of characteristic deflection obtained from the fourth measurement (-2.24) is slightly greater than the characteristic deflection before the guardrail removal. The reason for this is thought to be that since Span 2 includes the cantilever structure, it had some kind of influence on characteristic deflection. It is necessary, therefore, to continue various studies, including simulation analyses, on how to deal with spans that include cantilever structures.

Further discussions on the damage detection sensitivity of the system will be able to refer a Reference[58] which verified it by using 3D FEM analysis for damage detection accuracy.

4.3 Overview of Overseas Field Test

This section reports on the field test conducted in the city of Lisbon in Portugal as an overseas application of the bus monitoring system.

As part of a joint study made by Yamaguchi University and the Universidade Nova de Lisboa (NOVA), the field test was conducted with the cooperation of a group led by Prof. Vlter J. G. Lcio of NOVA. Figure 38 shows the fixed-route bus used in the experiment. The bus used in the field test is a fixed-route bus actually operating in Lisbon. The total weight is about 14 tons, and an under-rear-wheel-spring triaxial acceleration sensor was installed to the bus. Two bridges located near the university were chosen for the experiment. In this paper, the two bridges are referred to as "FCT-in" and "FCT-out." Table 11 and Figures 39(a) and (b) show the structural types, dimensions and appearances of the bridge, along with other general information. For the purpose of checking the coupling between the bus and the bridge of interest, a velocity sensor was installed in the middle section of each bridge to measure the velocity of the bus passing by on the bridge. The velocity sensor actually used in the field test is shown in Figure 40. At each bridge, measurement was conducted 15 times at a travel speed of 40 km/h.

Figures 41(a) and (b) compare the under-rear-wheel-spring acceleration of the bus and the girder midspan acceleration of the bridge measured when the bus crossed the two bridges. The under-rear-wheel-spring acceleration response of the bus here is the vertical acceleration data obtained from a triaxial acceleration sensor, and the center-of-bridge acceleration response of the bridge was obtained by converting the velocity data obtained from a velocity sensor by differentiating it once. At FCT-in, which is a continuous slab girder bridge as shown in Figure 39(a), the under-rear-wheel-spring acceleration response of the bus and the middle section acceleration response of the bridge are fairly similar as shown in Figure 41(a). At FCT-out, which is a cantilever bridge as shown in Figure 39(b), the under-rear-wheel-spring acceleration response and the bridge acceleration response were fairly similar when the bus was passing the center-of-cantilevered-span zone at the left as shown in Figure 41(b). When the bus was passing the center-of-bridge joint zone, however, the bridge showed a larger acceleration response to the extent that similarity with the under-rear-wheel-spring acceleration of the bus was no longer discernible. This is thought to be because the bus practically climbed over the central joint (i.e. cantilever joint) so that the vibration caused by the weight of the bus had a great influence on the bridge. Thus, it was found that the bus monitoring system can be applied to a continuous girder or simple girder bridge but cannot be applied to a cantilever bridge with a midspan joint like FCT-out.

This section shows characteristic deflection calculation results for FCT-in, which was thought to be a bridge of the type to which the bus monitoring system can be applied. As the first step, to determine the characteristic deflection of FCT-in, data for a period of about 1.8 seconds corresponding to the time the bus took to pass the middle section of the slab girder bridge (the encircled area in Figure 41(a)) were extracted, and deflection was calculated by integrating

acceleration twice. **Figure 42** shows examples of deflection estimates thus obtained. Showing estimated deflection on the vertical axis and time on the horizontal axis, **Figure 42** shows estimated deflection values obtained from the 15 measurements. As shown in **Figure 42**, data were extracted from the same section, and the obtained waveforms show similarity. Characteristic deflection is calculated by averaging estimated deflection values in this extraction section. **Table 12** lists the characteristic deflection values obtained from the 15 measurements. Thus, the average of the results of the 15 measurements that make characteristic deflection converge to a median is -4.10 mm (standard deviation: 0.63). It is important to continue to conduct more measurements by using this average as a criterion value and monitor changes in characteristic deflection over a long period of time.

As mentioned above, the field test results that have been obtained indicate that the proposed system can be applied to a continuous or simple girder bridge but cannot be applied to a cantilever bridge with a midspan joint like FCT-out. It is therefore necessary to conduct more field tests on other types of bridges. In view of the result of the test conducted in a country other than Japan, it has also been found that the proposed system can be used in other countries where conditions such as the instrumentation environment and vehicles used for measurement differ from those in Japan. The next task is to apply the proposed system to more bridges in other countries, conducting not only short-term but also long-term behavior measurements.

4.4 Summary

This paper has reported on the development of a new short and medium span bridge monitoring method using a fixed-route bus, presented field test results and discussed evaluation methods. Then, as a result of this study, this paper has shown that it may be possible to detect structural anomalies of bridges efficiently from changes in "characteristic deflection." On the other hand, it has also been found that characteristic deflection values calculated from measurement data varied widely. This is thought to be due to differences in the operating conditions of the bus. As the next step, it is necessary to conduct more measurements and investigate the effects of long-term operating conditions on results obtained, with the aim of improving characteristic deflection so as to reduce its variability and putting the proposed system to practical use as a long-term monitoring system.

The findings of this study are summarized below:

- 1) The development of a new short and medium span bridge monitoring system utilizing a publicly operated fixed-route bus and field test results have been reported, and evaluation methods have been discussed. As a result, it has been shown that it may be possible to detect structural anomalies of bridges through long-term measurement of characteristic deflection. It has also been found, however, that characteristic deflection values calculated from measurement data varied considerably. This is thought to be attributable to differences in the operating conditions of the bus. Changes in characteristic deflection have been observed for about three years through field testing. Since no sharp declining trend has been observed, it is necessary to continue long-term measurement.

- 2) In the field test carried out by using an out-of-service bridge being removed, a study was conducted on the influence(sensitivity) of artificial damage (bridge railing removal) on characteristic deflection. The experimental results for Span 3 of the bridge indicate that characteristic deflection is reasonably sensitive to declines in the flexural stiffness of the bridge structure. Since the characteristic deflection results for Span 2 show a considerable degree of variability, various studies including simulation analyses need to be conducted.

- 3) The field test of the bus monitoring system in an overseas environment showed that the system can be used without being affected by the measuring environment and the vehicle used. It has also

been found, however, that it may be difficult to apply the system to certain types of bridges such as cantilever bridges.

5. CONCLUDING REMARKS

Bridge health monitoring using the latest information technology and sensors will be becoming more challenging issues in the future for maintenance and rehabilitation of existing bridges. In this paper, although several methods are available for monitoring and assessment of existing short and medium span bridges, two advanced structural health monitoring systems were introduced.

In the first part of this paper, as one of the advanced data collecting system for experimental safety evaluation developed by German research team, a special designed in-situ loading vehicle, BELFA which is available for assessment quantitatively of load carrying capacity of the target bridge was discussed as a structural health monitoring system at the bridge site. The proposed method was applied to a few actual existing concrete bridges in a rural area of Germany as a specific example to verify its effectiveness. As the results, it will be able to make a rational periodical health monitoring system for existing short and medium span bridges, and then the method helps bridge administrators to establish the rational maintenance strategies.

In the second part, as one solution to the problem for condition assessment of existing short and medium span reinforced and prestressed concrete bridges, a new long term monitoring method using a public bus as part of a public transit system (bus monitoring system) was introduced, along with safety indices, namely, characteristic deflection, which is relatively free from the influence of dynamic disturbances due to such factors as the roughness of the road surface, and a structural anomaly parameter. Moreover, the details of not only how to assess the bridge condition by public bus vibration measured in operating on Ube City bus network as a specific example for verify the system but also what kind of consideration we need to apply the system to existing bridges in overseas country were discussed.

The author would hope that this paper would be of some help in this field in the future.

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Figure 1. Basic components of a typical SHM system [8]

Figure 2. Basic elements of BELFA in working position[4]

Table 1. Essential data of BELFA[4]

Length in transportation mode:	22.5 m
Max. length in working position:	35.5 m
Max. distance between supports	18.0 m
Total weight	approx. 84.5 t
Max. test load (without additional dead freight)	approx. 750 kN
Max. permissible test load	1500 kN

Figure 3(a). Arrival of BELFA in transportation mode[4]

Figure 3(b). Crossing the bridge placing BELFA[4]

Figure 3(c). BELFA in operation mode, supported at both bridge bearings[4]

Figure 4. View of in-situ loading test by BELFA for an existing bridge[4]

Figure 5. Cross section of a target bridge, BS-bridge (midspan) [4]

Figure 6. An example of load deflection curves at midspan by BELFA under four various conditions[4]

Figure 7. Side view and midspan cross section of short span bridge, MO-bridge[4]

Figure 8. View of in-situ monitoring by BELFA for short span bridge in rural area[4]

Figure 9. Reduction path of safety level and coverage of proposed system

Figure 10. Concept of the bus monitoring system

Figure 11. Analysis flow for bus monitoring system

Figure 12. Simple spring-mass model of the bridge-vehicle interaction system

Figure 13. Input to the vehicle system and over-spring and under-spring deformations

Figure 14. Over-spring/under-spring/bridge substructuring scheme

Figure 15. “Characteristic deflection” calculation flow

Figure 16. Examples of characteristic waveforms of estimated deflection

Figure 17. Example of determination of acceleration data extraction range

Figure 18. Example of estimation of deflection

Figure 19. Number of bridges managed by Ube-city located on bus routes run by Ube-city Transportation Bureau(UTB)

Figure 20. Percentage of bridges 50 years or older in all bridges in Ube-city 20 years from now

Table 2. Data on bridges selected for long-term monitoring

Bridge name	Completed in	Type of superstructure			Span length (m)	Bridge length (m)
Jase Bridge	1976	Span No.	Start point side 1	Prestressed concrete slab bridge (pretensioned slab)	18.0	85.0
			2	Prestressed concrete slab bridge (pretensioned slab)	16.0	
			3	Prestressed concrete slab bridge (pretensioned slab)	18.0	
			4	Prestressed concrete slab bridge (pretensioned slab)	14.0	
			End point side 5	Prestressed concrete slab bridge (pretensioned slab)	19.0	
Shiratsuchi Daini Bridge	1933 (estimated)	Span No.	Start point side 1	Reinforced concrete (T-girder)	7.0	15.0
			End point side 2	Reinforced concrete (T-girder)	7.0	
Shingondai Bridge	June 1998	Single-span prestressed concrete girder bridge (Bi-prestressing method)			22.4	23.6

(a) Shingondai Bridge (single-span bridge)

(b) Shiratsuchi Daini Bridge (two-span bridge)

(c) Jase Bridge (five-span bridge)

Figure 21. General views of Ube-city bus route bridges selected for long-term monitoring

Table 3. Specifications of bus (vehicle) used for long-term monitoring

Item	Specifications
Net vehicle weight	8,130kg
Gross vehicle weight	11,485kg
Front axle weight	2,730kg
Rear axle weight	5,400kg
Wheel base	4.4m

Table 4. Specifications of three-axis acceleration sensor used for long-term monitoring (vehicle side)

Name: Fuji Ceramics SA11ZSC-TI (Three-axis piezoelectric acceleration transducer with built-in amplifier)	
Charge sensitivity	1mV/m/s ²
Frequency range	1 - 8,000 Hz
Resonant frequency	35 kHz or higher
Maximum measurable acceleration	4,000 m/s ²
Maximum allowable acceleration	30,000 m/s ² or higher
Power supply for built-in amplifier	21 - 24 V/0.5 - 10 mA
Temperature range	-50 – +110°C
Dimensions	14.2 × 14.2 × 14.2 mm
Mass	Approx. 11.1 g

Figure 22. General view of the bus (vehicle) used for the Bus Monitoring System

(a) Acceleration sensor installed to the rear wheel of the bus

(b) Wiring routed into the bus

Figure 23. Installation of acceleration sensor to the bus (vehicle)

Figure 24. Configuration of the on-board measurement system of the Bus Monitoring System

Figure 25. Measurement in progress in the bus and the measuring equipment used (in the back of the bus)

Figure 26. Acceleration sensor location at the bridge and the route and direction of movement of the bus

Figure 27. Example of acceleration response waveform in the midspan zone of the bridge

Figure 28. Comparison between midspan acceleration response waveform and rear-wheel-under-spring acceleration response waveform and results of FSWT based time–frequency space analysis

Figure 29. Differences in standard deviation of characteristic deflection among data sections (Shingondai Bridge: Tokonami → Nishikiwa (direction of vehicle movement))

Figure 30. Examples of measured values of characteristic deflection and simple moving averages

Table 5. Serious deterioration (damage) of “Shingondai Bridge (PC girder bridge)” used in vehicle-induced vibration simulation and changes in characteristic deflection

Structural soundness of bridge	Decrease in prestressing force	Ratio of geometrical moment of inertia relative to 0% reduction	Ratio of characteristic deflection relative to 0% reduction
Sound	0 %	1.0	1.0
Deterioration Phase 1	50 %	0.52	1.93
Deterioration Phase 2	90 %	0.35	2.86

(a) Tokonami→Nishikiwa Gakkomae

(b) Nishikiwa Gakkomae→Tokonami

Figure 31. Example of changes in characteristic deflection from 2010 to 2013 (Shingondai Bridge) and comparison with the serious deterioration (damage) criteria (calculated values) shown in Table 5

Table 6. Conditions (data restrictions) for correlation coefficient calculation by bridge

Bridge name	Speed (km/h)	Rainfall(mm)	Temperature(°C)	Oncoming traffic (vehicles)	Number of persons on vehicle (persons)
Shingondai Bridge	40–50	0	20–30	0	5–15
Shiratsuchi Daini Bridge	40–50	0	20–30	0	5–15
Jase Bridge	45–55	0	20–30	0–1	No restriction

Table 7(a). Correlations between characteristic deflection and bus operating conditions by bridge
(correlation with vehicle speed/rainfall/temperature)

Bridge name	Direction of movement	Span	Speed	Rainfall	Temperature
Shingondai Bridge	Toko → Nishi	–	– 0.162	0.240	0.135
			Little correlation	Weak positive correlation	Little correlation
	Nishi → Toko	–	– 0.257	0.151	– 0.337
			Weak negative correlation	Little correlation	Weak negative correlation
Shiratsuchi Daini Bridge	Nishi → Yoshi	A	– 0.014	– 0.095	0.005
			Little correlation	Little correlation	Little correlation
		B	0.022	– 0.201	– 0.182
			Weak positive correlation	Weak negative correlation	Little correlation
	Yoshi → Nishi	B	– 0.434	0.317	– 0.507
			Negative correlation	Weak positive correlation	Negative correlation
		A	– 0.058	– 0.008	– 0.136
			Little correlation	Little correlation	Little correlation
Jase Bridge	Sho → Kin	A	– 0.192	0.091	0.004
			Little correlation	Little correlation	Little correlation
		B	0.026	0.117	0.071
			Little correlation	Little correlation	Little correlation
		C	0.087	– 0.005	– 0.044
			Little correlation	Little correlation	Little correlation
		D	0.159	0.124	0.206
			Little correlation	Little correlation	Weak positive correlation
		E	0.095	0.062	0.186
			Little correlation	Little correlation	Little correlation
	Kin → Sho	E	– 0.270	– 0.117	– 0.135
			Weak negative correlation	Little correlation	Little correlation
		D	0.161	– 0.234	0.088
			Little correlation	Weak negative correlation	Little correlation
		C	– 0.348	0.046	– 0.144
			Weak negative correlation	Little correlation	Little correlation
		B	– 0.332	0.053	0.309
			Weak positive correlation	Little correlation	Weak positive correlation
		A	– 0.328	0.156	0.052
			Weak negative correlation	Little correlation	Little correlation

Table 7(b). Correlations between characteristic deflection and bus operating conditions by bridge (correlation with number of oncoming vehicles/number of persons on bus)

Bridge name	Direction of movement	Span	Oncoming traffic	Number of persons on bus
Shingondai Bridge	Toko → Nishi	–	– 0.059	0.205
			Little correlation	Weak positive correlation
	Nishi → Toko	–	– 0.150	– 0.101
			Little correlation	Little correlation
Shiratsuchi Daini Bridge	Nishi → Yoshi	A	0.124	– 0.097
			Little correlation	Little correlation
		B	0.222	0.044
			Weak positive correlation	Little correlation
	Yoshi → Nishi	B	– 0.217	– 0.152
			Weak negative correlation	Little correlation
		A	0.148	– 0.011
			Little correlation	Little correlation
Jase Bridge	Sho → Kin	A	0.099	0.263
			Little correlation	Weak positive correlation
		B	– 0.266	– 0.270
			Weak negative correlation	Weak negative correlation
		C	0.024	0.018
			Little correlation	Little correlation
		D	0.328	0.081
			Weak positive correlation	Little correlation
		E	0.308	0.031
			Weak positive correlation	Little correlation
	Kin → Sho	E	0.013	– 0.044
			Little correlation	Little correlation
		D	0.088	0.469
			Little correlation	Positive correlation
		C	0.036	– 0.322
			Little correlation	Weak negative correlation
		B	– 0.009	0.366
			Little correlation	Positive correlation
		A	– 0.184	– 0.030
			Little correlation	Little correlation

Table 8. Correspondence between the range of correlation coefficients and correlations (expressed in words)

Coefficient of correlation	Correlation
0.0 – ± 0.2	Little correlation
± 0.2 – ± 0.4	Weak correlation
± 0.4 – ± 0.7	Correlated
± 0.7 – ± 0.9	Strong correlation
± 0.9 – ± 1.0	Very strong correlation

Table 9. Number of measurement data sets

Bridge name	Direction of movement	Number of measurement data sets
Shingondai Bridge	Toko → Nishi	80 sets
	Nishi → Toko	
Shiratsuchi Daini Bridge	Nishi → Yoshi	77 sets
	Yoshi → Nishi	
Jase Bridge	Sho → Kin	66 sets
	Kin → Sho	64 sets

Table 10. Calculated values of characteristic deflection by bridge/span

Bridge name	Direction of movement	Span	Characteristic deflection(mm)	
			Average	Standard deviation
Shingondai Bridge	Toko → Nishi	–	– 5.218	1.733
	Nishi → Toko	–	– 2.909	1.231
Shiratsuchi Daini Bridge	Nishi → Yoshi	A	– 2.731	1.071
		B	– 2.030	0.868
	Yoshi → Nishi	B	– 1.577	0.727
		A	– 2.439	1.021
Jase Bridge	Sho → Kin	A	– 2.153	0.608
		B	– 1.910	0.533
		C	– 2.017	0.651
		D	– 2.085	0.657
		E	– 2.467	0.669
	Kin → Sho	E	– 1.423	0.628
		D	– 1.499	0.651
		C	– 1.131	0.547
		B	– 1.164	0.579
		A	– 1.532	0.554

Figure 32. Characteristic deflection values obtained by applying the simple moving average method to four-year monitoring data (Shingondai Bridge).

(a) Nishikiwa Gakkomae → Yoshida (Span A)

(b) Yoshida → Nishikiwa Gakkomae (Span A)

(c) Nishikiwa Gakkomae → Yoshida (Span B)

(d) Yoshida → Nishikiwa Gakkomae (Span B)

Figure 33. Characteristic deflection values obtained by applying the simple moving average method to four-year monitoring data (Shiratsuchi Daini Bridge)

Figure 34. Sightseeing bus used for the field test

Figure 35. Side view of the out-of-service bridge used for field testing

(a) Before guardrail removal

(b) After guardrail removal

Figure 36. Bridge view before and after guardrail removal

Figure 37. Changes in characteristic deflection due to guardrail removal

Figure 38. Fixed-route bus used and sensor location

Table 11. Bridge data

Bridge name	Type of superstructure				Bridge length (m)	
FCT-in	Span number	Start point	1	3-span continuous prestressed concrete slab girder bridge	16.00	68.13
			2		36.13	
		End point	3		16.00	
FCT-out	Span number	Start point	1	Reinforced concrete T-girder cantilever bridge	10.65	49.04
			2		13.87	
			3		13.87	
		End point	4		10.65

(a)FCT-in

(b)FCT-out

Figure 39. Fixed-route bus used and sensor location

Figure 40. Velocity sensor

(a)FCT-in

(b)FCT-out

Figure 41. Comparison of under-rear-wheel-spring and girder-midspan acceleration responses during bridge crossing

Figure 42. Deflection estimated from under-rear-wheel-spring acceleration response of bus (FCT-in)

Table 12. Characteristic deflection
(FCT-in)

No.	Characteristic deflection (mm)
1	-4.51
2	-4.06
3	-4.37
4	-4.26
5	-4.65
6	-4.67
7	-3.72
8	-2.97
9	-4.87
10	-4.23
11	-3.33
12	-4.83
13	-4.44
14	-3.43
15	-3.11
AVERAGE	-4.10